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BUREAU OF PUBLIC ROADS

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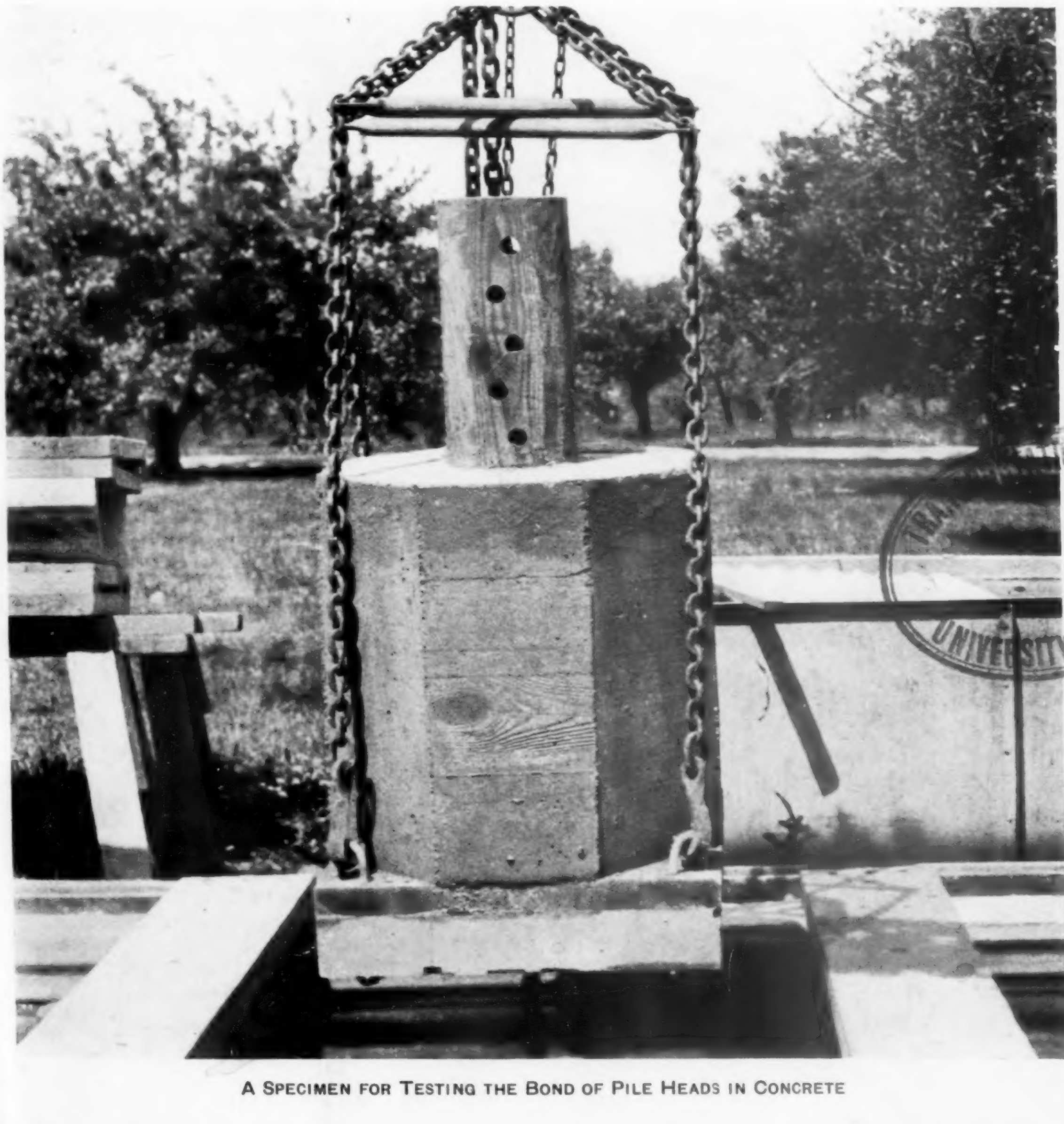

A SPECIMEN FOR TESTING THE BOND OF PILE HEADS IN CONCRETE

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FOUNDATION PILE-HEAD BOND AND ANCHORAGE TESTS

BY THE DIVISION OF TESTS, U. S. BUREAU OF PUBLIC ROADS

Reported by GEORGE W. DAVIS, Assistant Engineer of Tests

THE bond strength developed between the heads of timber piles and the concrete in which they are encased is of importance in bridge construction in connection with the design of concrete foundation seals for cofferdams. These seals, if they are to resist the hydrostatic pressure by their weight alone, must have a depth of about four-tenths of the hydrostatic head. If it can be assumed with certainty that a bond strength of appreciable magnitude will be developed between the pile heads of the foundation and the concrete seal in which they are encased, the depth of seal required will not be as great as would otherwise be the case, thus reducing the amount of concrete which must be cast under water. There are, of course, numerous applications to engineering design of this bonding property of concrete to timber pile heads other than that just described.

AN UNTURNED PILE SECTION

The series of tests by the Milwaukee Sewerage Commission[1] has shown that bond strengths of some magnitude may be expected but this investigation was of limited scope and practically no information, aside from that which it developed, appears to exist.

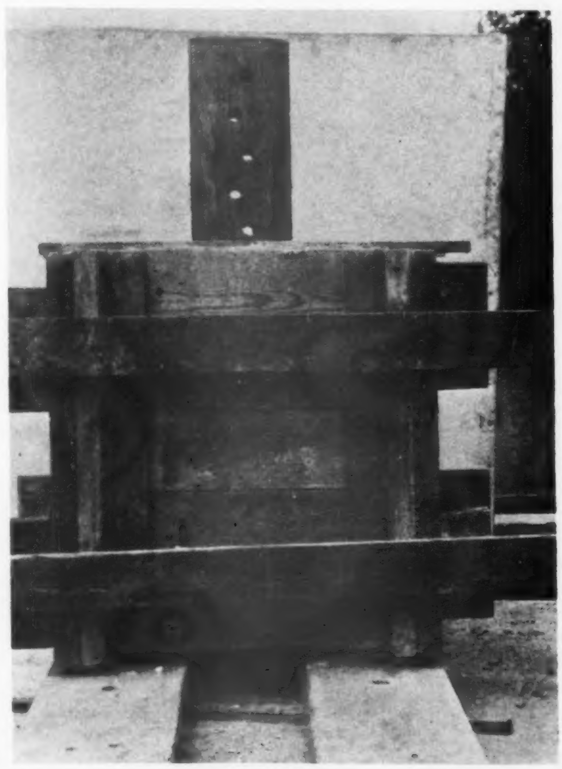
Many bridges in the United States, particularly those in the region of the Gulf coast, must be designed to withstand the action of hurricanes and it is sometimes necessary to reinforce the piers of such structures to resist the overturning forces developed by high winds. In such cases the effective anchorage of steel reinforcement to timber foundation piles is of importance in the design of substructures.

The limited investigation reported in this paper was conducted to add to the meager information available regarding the bond strength of timber piles in concrete and to investigate the effectiveness of various types of anchorage of reinforcement to timber piles.

SCOPE OF TESTS OUTLINED

The specimens used for determining the bond strength developed between the timber piles and concrete consisted of short sections of piles embedded in concrete blocks. The pile sections were of southern pine, 37 inches in length and varying from 7¾ to 11 inches in diameter. The timber was of rather inferior grade, as is indicated by the fact that representative pieces showed an average of about eight annual rings per inch in the outer 3 inches of the stick.

The pile sections were thoroughly soaked by immersion in water for several days and then, while wet, were encased for 20 inches of their length in concrete blocks 24 inches in depth. These blocks were octagonal in form with a minimum diameter of 24 inches and were reinforced with a spiral of ½-inch round steel bar. Details of the bond-test specimens are shown in Figures 1 to 3 and in the accompanying illustrations. The concrete was machine mixed and was of a plastic but not sloppy consistency. The proportions used were 1:2:4 by volume and the aggregates were sound and well graded local materials. All but four of the pile sections were lathe turned to a constant diameter for their entire length so that the bond strength measured might be that developed under the most adverse surface condition of the pile itself. The four unturned



PILE SECTION ENCASED IN CONCRETE BEFORE THE REMOVAL OF THE FORMS

pile sections, all of which had a very small taper, were peeled and roughly trimmed but otherwise were treated as were the other plain specimens in order that a comparison might be had of the relative bond developed by the turned and unturned sections.

[1] LUNDAHL, R. R., BOND STRENGTH OF WOOD PILES IN CONCRETE. Transactions of the Amer. Soc. of Civil Engineers, 1923, Vol. LXXXVI, p. 268.

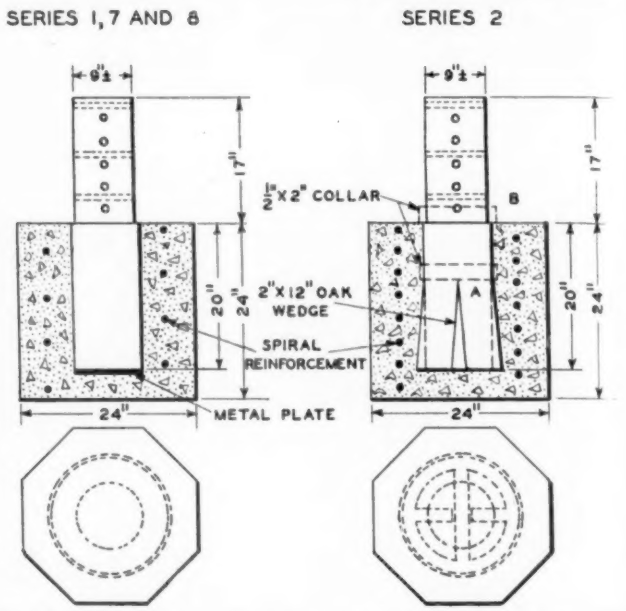
Anchorage Specimens Showing Pile Ends Flush with Surface of Concrete (Top Picture) and Specimens Ready for Curing Under Water

The arrangement for the anchorage tests consisted of pile sections 27 inches long embedded for their full length in concrete cylinders 30 inches in depth. The quality of the concrete was the same as that used in the bond tests and the cylinders were reinforced with ½-inch round spiral reinforcement. The details of the specimens for the anchorage tests are shown in Figure 4.

All of the concrete encasement for both bond and anchorage tests, with the exception of that for series 8, was cast in the air, protected with wet burlap, and allowed to harden for 24 hours. The specimens were then immersed in water and allowed to cure for 7 or for 21 days before testing, and in all cases were tested directly after being taken out of the immersion tank.

The concrete encasement for the bond specimens of Series 8 was cast under water, using a small tremie, and the specimens remained under water for 21 days, at which age they were tested.

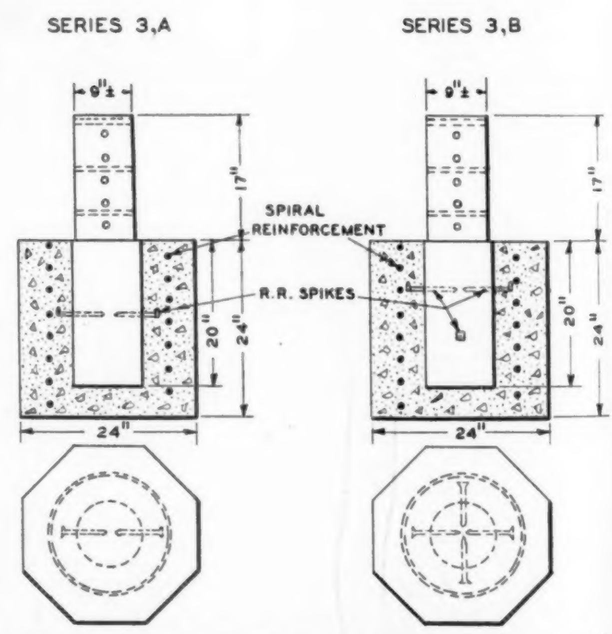
Neither the concrete in the specimens cast in air nor that in the specimens cast in water is believed to have been truly representative of concrete as it would ordinarily be placed in foundation seals cast under water.



SERIES 1.—PLAIN BOND, CONCRETE PLACED IN AIR
SERIES 7.—SAME AS SERIES 1, EXCEPT END BOND PREVENTED BY THIN METAL PLATE
SERIES 8.—SAME AS SERIES 1, EXCEPT CONCRETE PLACED UNDER WATER

SERIES 2.—PILE HEAD EXPANDED WITH OAK WEDGES. STEEL COLLAR PLACED IN PILE AT A, WHILE WEDGES WERE DRIVEN, THEN MOVED TO B WHILE CONCRETE WAS PLACED, AND REMOVED ENTIRELY BEFORE TESTING

Fig. 1.—Details of Specimens of Series 1, 7, 8, and 2

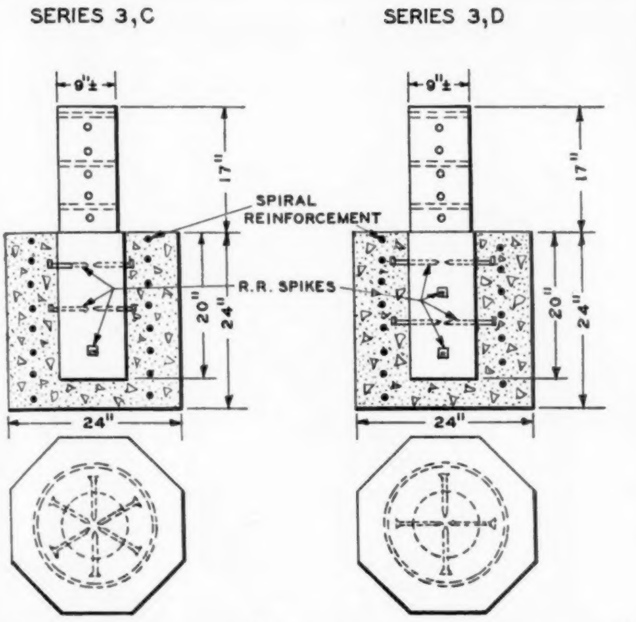


SERIES 3, A.—TWO RAIL SPIKES ⅝ BY 6 INCHES, DRIVEN AS SHOWN

SERIES 3, B.—FOUR RAIL SPIKES ⅝ BY 6 INCHES, DRIVEN AS SHOWN

Fig. 2.—Details of Test Specimens for Bond Tests of Series 3,A and 3,B

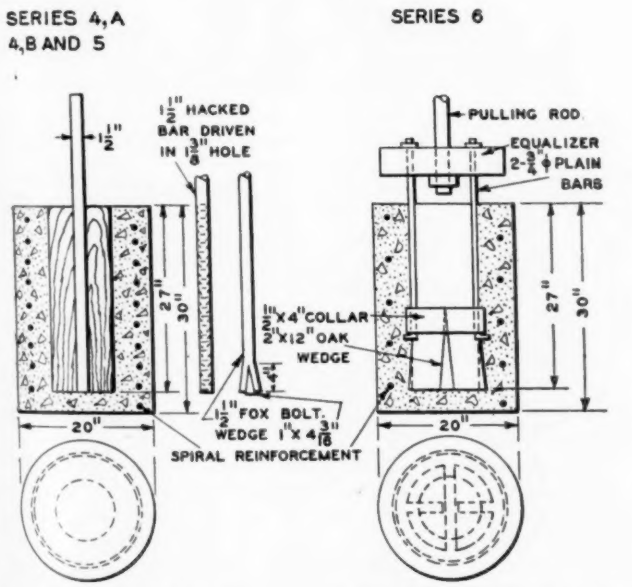
The concrete cast in the air may be considered as representative of concrete of the same proportions in construction which is placed "in the dry" but was probably better compacted and therefore of a greater density and strength than concrete as normally placed under water. On the other hand, the specimens cast under water



SERIES 3. C.—SIX RAIL SPIKES ⅝ BY 6 INCHES, DRIVEN AS SHOWN

SERIES 3. D.—EIGHT RAIL SPIKES ⅝ BY 6 INCHES, DRIVEN AS SHOWN

FIG. 3.—DETAILS OF SPECIMENS FOR BOND TESTS OF SERIES 3,C AND 3,D

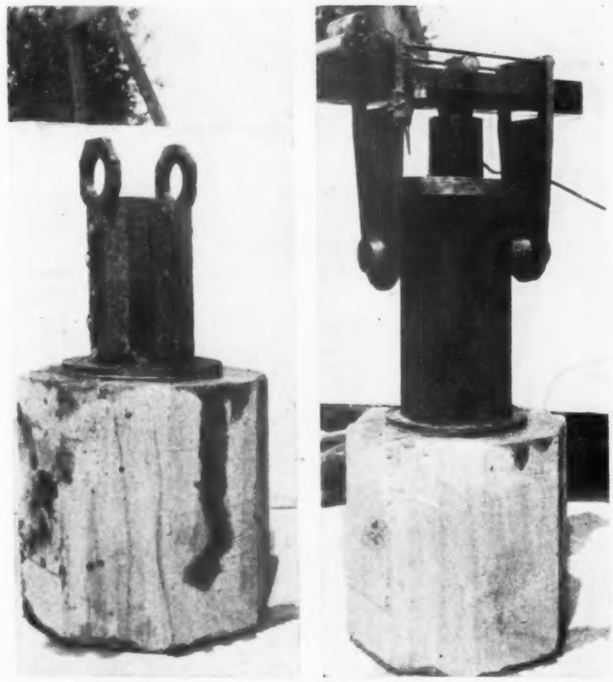


SERIES 4. A.—HACKED BAR USED
SERIES 4. B.—SMOOTH BAR USED
SERIES 5.—FOX BOLT USED

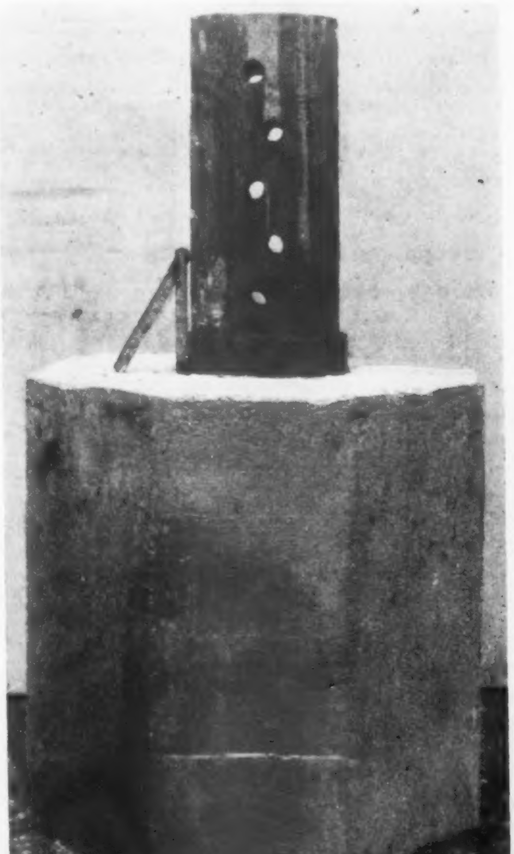
SERIES 6.—PILE HEAD EXPANDED WITH OAK WEDGES. REINFORCING BARS HOOKED UNDER STEEL COLLAR

FIG. 4.—DETAILS OF SPECIMENS FOR ANCHORAGE TESTS OF SERIES 4, 5, AND 6

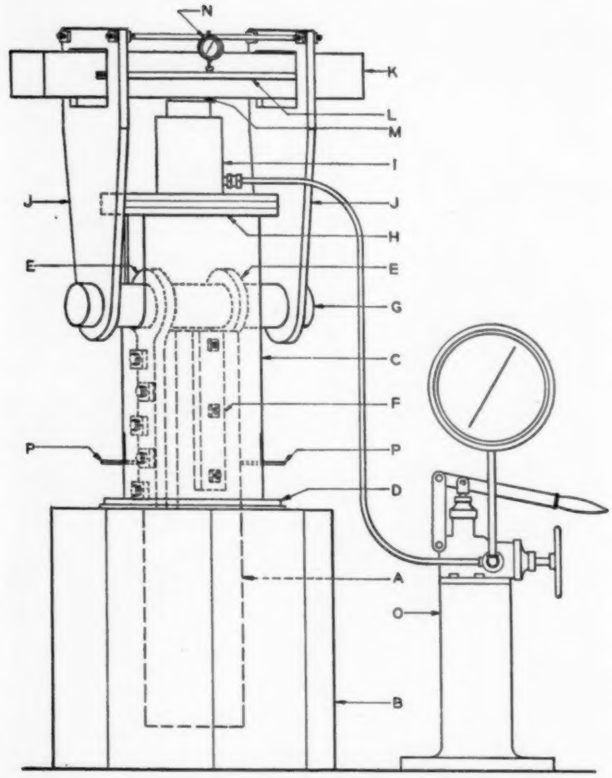
are believed to have been inferior to the concrete which normally would be encountered in foundation seals since it was not as rich in cement as the mixtures ordinarily employed for this purpose, and, since the specimens were of shallow depth and were cast in water which barely covered them, they were not subjected to the pressure of superimposed water and concrete which, in actual construction, would tend to compact the concrete around the foundation pile heads.



BOND-TEST SPECIMEN SHOWING EYEBARS AND SIDE CLAMPS ATTACHED AND APPARATUS SET UP READY FOR TEST

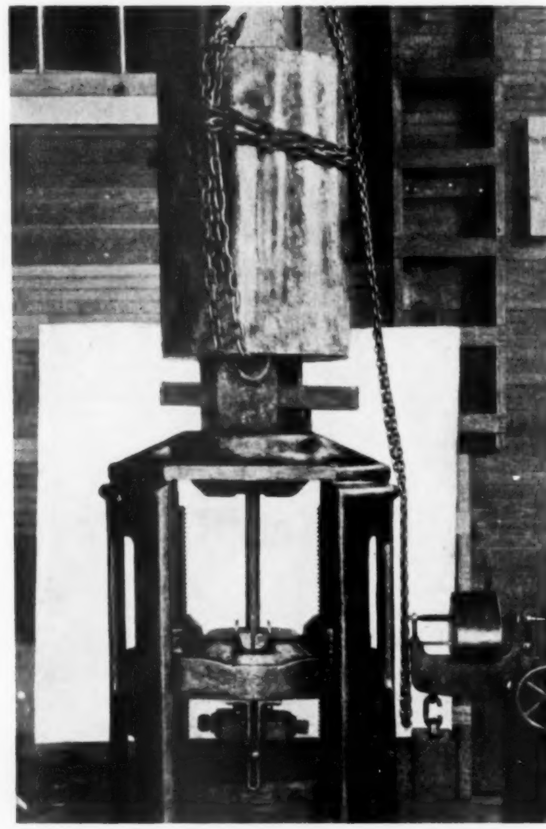


BOND SPECIMEN AFTER TEST. PILE PULLED 2 INCHES FROM ITS ORIGINAL POSITION IN THE CONCRETE CASING



A.—PILE SECTION
B.—CONCRETE BLOCK
C.—STEEL CYLINDER
D.—STEEL RING RESTING ON RUBBER PAD
E.—EYEBARS
F.—SIDE CLAMPS
G.—PIN
H.—COVER PLATE
I.—HYDRAULIC JACK (5-INCH RAM)
J.—SIDE PLATES OF PULLING YOKE
K.—CALIBRATED BEAM
L.—RIDER BAR
M.—KNIFE-EDGE
N.—AMES DIAL
O.—PUMP FOR JACK
P.—STEEL PINS FOR MEASURING SLIP

FIG. 5.—DETAILS OF BOND-TESTING APPARATUS



A SPECIMEN OF SERIES 6 READY FOR TEST. NOTE THE EQUALIZER BETWEEN THE TWO REINFORCING BARS

SPECIAL APPARATUS DEVISED FOR BOND TESTS

In order to pull the piles from the surrounding concrete casings it was necessary to devise and build the special apparatus shown in Figure 5. This device consists of a hydraulic jack attached to the protruding end of the pile by suitable mechanical linkage and reacting against the concrete casing around the pile. When pressure is exerted by the jack the fibers of the pile are subjected to tension as would be the case in an actual structure.

Referring to Figure 5, the pile, A, is shown encased in the concrete block, B. The two eyebars, E, are bolted to opposite sides of the squared, protruding end of the pile, which is restrained from splitting by a pair of side plates, F, bolted to the other two sides of the pile. Through the eyebars is a large steel pin, G, and over the outer ends of this pin are two side plates, J, which complete the link between the pile and the calibrated crossbeam, K. This beam is supported at its center by the head of the hydraulic jack, I, which is in turn supported by a steel cylinder, C, surrounding but not in contact with the pile, and bearing against the concrete block, through an annular steel plate and rubber pad, D.

Pressure was supplied to the jack by an oil pump, O, operated by hand. The magnitude of the tensile load being applied to the pile was determined by the deflection of the calibrated cross beam, K, which was provided with knife-edges at the load-reaction points. The deflection was measured with a micrometer dial, N, mounted on the beam at the mid-point. The movable stem of this dial rested against a free beam, L, mounted along the axis of the calibrated beam but not in contact with it and supported by the plates, J. The stiffness of the calibrated beam was such that a reading of one division on the micrometer dial indicated an applied load of 317 pounds.

Slip of the pile in the concrete block was determined by measuring the vertical movement of pins, P, with respect to the upper surface of the concrete block. These pins were driven into the pile through slots in the wall of the steel cylinder, after the cylinder had been placed in position.

In the anchorage tests the specimens were inverted over the head of a 200,000-pound universal testing machine as shown in Figure 6 and the reinforcement attached to the moving head of the machine, either directly where a single rod was being pulled or by an equalizing bar and a single connection where two rods were anchored in the same specimen.

TEST SPECIMENS DESCRIBED IN DETAIL

Figures 1 to 4 and the following outline give full details regarding the various specimens and the ages at which they were tested.

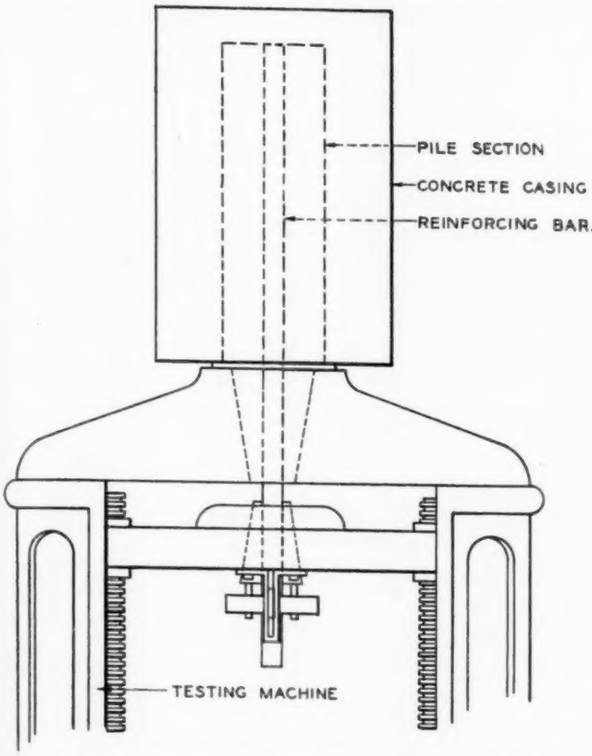
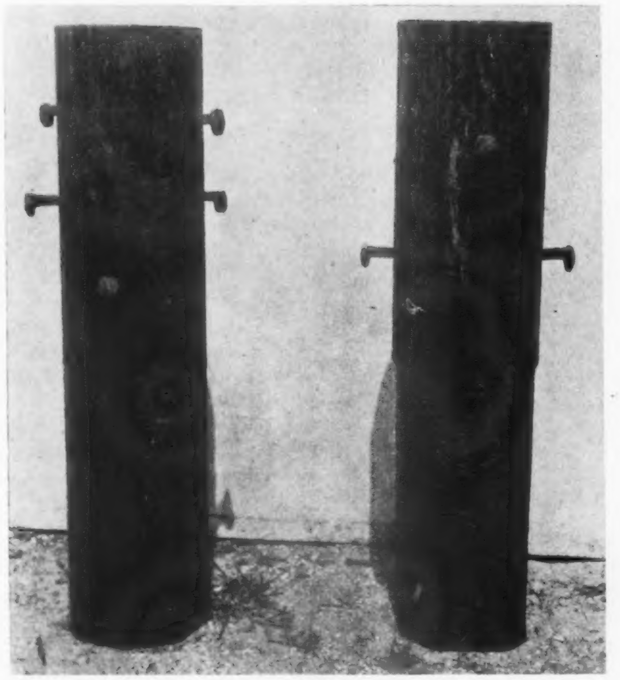


FIG. 6.—APPARATUS USED IN ANCHORAGE TESTS

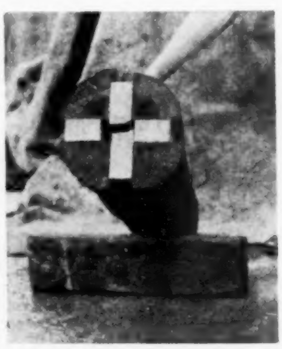


PILE SECTION OF SERIES 3,B (RIGHT) AND OF SERIES 3, C (LEFT)

BOND TESTS

Series 1.—Twelve turned and four unturned pile sections embedded in concrete without artificial bond. Six of the turned and two of the unturned, or half of each type of pile sections, were tested at the age of 7 days and the other half at the age of 21 days.

Series 2.—Six turned pile sections. An attempt was made to increase the bond by expanding the end of the pile on two diameters at right angles to each other by driving oak wedges 2 inches thick and 12 inches long into sawed slots. The pile sections were prevented from splitting completely by a steel collar placed at the end of the saw slots. This collar was removed after the wedges had been driven and before the concrete casing was cast and another collar was placed just above where the surface of the concrete would come when placed. This latter collar was removed before test. All of these specimens were tested at the age of 21 days.



PILE SECTION OF SERIES 2 BEFORE ENCASEMENT

Series 3,A.—Three turned pile sections into which two railroad spikes were driven on opposite sides and 10 inches from the end of the pile. The spikes were driven 4 inches into the pile and 2 inches extended into the surrounding concrete. These specimens were tested at the age of 21 days.

Series 3,B.—Two pile sections as in Series 3,A, except that four spikes were driven into the pile at distances of 6½ inches and 13 inches from the end. These specimens were tested at the age of 21 days.

Series 3,C.—Two pile sections as in Series 3,A, except that six spikes were driven into the pile at points 120° apart on the circumference and at distances of 5, 10, and 15 inches from the end of the pile. These specimens were tested at the age of 21 days.

Series 3,D.—Two pile sections as in Series 3,A, except that eight spikes were driven into the pile in opposing pairs at distances of 4, 8, 12, and 16 inches from the end of the piles. These specimens were tested at the age of 21 days.

Series 7.—Six turned pile sections identical with Series 1 except that a thin metal plate was tacked to the end of the pile section so as to prevent any bonding of the concrete to the end of the pile. These specimens were tested at the age of 21 days.

Series 8.—Six turned pile sections which were encased in concrete, while submerged, by the use of a small tremie. These specimens were tested at the age of 21 days.

ANCHORAGE TESTS

Series 4,A.—Four specimens. In this group round steel bars 1½ inches in diameter were scored or hacked with a cold chisel at 1-inch intervals along four sides or elements of the bar for a distance of 27 inches. These bars were then driven for 27 inches into 1⅜-inch auger holes bored through the longitudinal axis of the pile. In two cases the bars were driven before the pile was encased in concrete. These two specimens were tested at the age of 7 days. The other two piles were first encased, the bars were driven 14 days later, and the specimens were tested at the age of 21 days.

Series 4,B.—Four specimens similar to those in Series 4,A except that smooth round bars were used. Two rods were driven 14 days and two 20 days after the piles were encased in concrete and all specimens were tested at the age of 21 days.

Series 5.—Four specimens. Fourteen days after the pile sections were encased a "fox bolt" 1½ inches in diameter, was driven into a 1⅜-inch hole 27 inches deep bored along the axis of each pile. All specimens were tested at the age of 21 days.

Series 6.—Four specimens. Each pile section was saw slotted for 12 inches of its length across two diameters of the pile. At the end of these slots a heavy split steel collar was clamped around the pile. Two ¾-inch round bars were hooked under this collar on opposite sides of and extended upward along the surface of the pile section. The head of the pile was then expanded with oak wedges as in the bond tests of series 2 and the whole encased in concrete. These specimens were all tested at the age of 21 days.

PILE SECTION OF SERIES 6 READY FOR ENCASEMENT

BOND STRENGTH NOT MATERIALLY INCREASED BY WEDGES AND SPIKES

The data obtained during these tests consist principally of the measured load when the first slip occurred, certain dimensional information, and in some cases observations relative to the behavior of the particular specimens. These data are summarized in Tables 1 and 2. In computing the unit bonds the total load at slip actually indicated by the micrometer dial was divided by the area of the embedded cylindrical surface (end area excluded). The dead weight of the mechanical linkage was a negligible percentage of the total load, so was not deducted. The end area was not included in the bonded area since the tests of Series 7 showed no evidence of appreciable end bond.

The first observation to be made from an examination of the data is that a definite and appreciable bond existed in all cases. Six turned specimens tested at the age of 7 days showed an average bond of 42.5 pounds per square inch and a minimum bond of 33 pounds per square inch. Five turned specimens tested at the age of 21 days showed an average bond of 62.6 pounds per square inch and a minimum bond of 55 pounds per square inch. Unturned specimens tended to give somewhat higher bond values but showed considerably greater variation between specimens as would naturally be expected.

The turned specimens of Series 1 indicate a consistent tendency for the bond strength to increase considerably from 7 to 21 days. The data for the unturned specimens do not show this tendency, but this is believed to be due to the small number of specimens of this type which were tested at each age. The variability of the unturned specimens would thus greatly affect average values and cause them to be of little value for this comparison.

In Series 2 and 3 the attempt was made to increase the holding power by artificial means, and it is evident that the initial slip occurred at practically the same

TABLE 1.—*Data pertaining to the pile-head bond tests*

Series	Specimen No.	Artificial bond	Age	Diameter	Area: Side	Area: End	Load at initial slip	Unit load at initial slip	Remarks	Load after 2-inch slip: Lbs.	Load after 2-inch slip: Lbs. per sq. in.
			Days	*In.*	*Sq. in.*	*Sq. in.*	*Pounds*	*Lbs. per sq. in.*			
1	19	None	7	9⅛	573.3	65.4	18,730	33			
	28	...do	7	9⅜	589.0	69.0	27,610	47			
	29	...do	7	9	565.5	63.6	25,397	45			
	31	...do	7	8⅜	526.2	55.1	27,300	52			
	36	...do	7	9⅛	573.3	65.4	25,714	45			
	37	...do	7	9⅛	573.3	65.4	19,050	33			
								42.5	Average.		
	21	...do	21	8⅛	510.5	51.8	33,016	65			
	23	...do	21	8	502.7	50.3	33,655	67			
	24	...do	21	9¼	581.2	67.2	31,750	55			
	25	...do	21	8⅞	557.6	61.9	38,095	68			
	33	...do	21	9⅜	589.0	69.0	34,285	58			
	[1] 34	...do	21	9¼	581.2	67.2	25,715	[1] 44			
								50.5	Average of all specimens.		
								62.6	Average, excluding No. 34.		
	20	...do	7	10¼	644.0	82.5	61,905	96	Unturned.		
	22	...do	7	9¼	581.2	67.2	35,873	62	Do.		
								79	Average.		
	27	...do	21	9	565.5	63.6	27,936	49	Unturned.		
	[2] 30	...do	21	11	691.2	95.0	43,808	[2] 63	Do.		
								56	Average.		
2	12	Wedges	21	8½	534.1	56.7	33,020	62		71,437	134
	14	...do	21	8⅝	541.9	58.4	34,925	64		98,425	182
	9	...do	21	8⅝	541.9	58.4	34,285	63		63,490	117
	10	...do	21	8¾	549.8	60.1	34,920	63.5		63,650	115.8
	11	...do	21	8⅞	557.6	61.9	40,635	73		66,667	120
	13	...do	21	8¼	518.4	53.5	33,333	64		77,777	150
								64.9	Average.		
3, A	26	2 railroad spikes.[3]	21	8½	534.1	56.7	36,200	68			
	32	...do	21	8¼	518.4	53.5	29,520	57			
	35	...do	21	9	565.5	63.6	41,270	73			
								66	Average.		
3, B	44	4 railroad spikes.[3]	21	8⅛	510.5	51.8	30,158	59			
	45	...do	21	9	565.5	63.6	20,635	37	Poor pile (soft).		
3, C	46	6 railroad spikes.[3]	21	9¼	581.2	67.2	31,746	55			
	47	...do	21	8¾	549.8	60.1	14,286	26	Poor pile (soft).		
3, D	57	8 railroad spikes.[3]	21	9¼	581.2	67.2	34,920	60			
	58	...do	21	9¼	581.2	67.2	44,444	76	Concrete cracked.		
								56.8	Average of all specimens.		
								64	Average, excluding Nos. 45, 47.		
7	38	None	21	8	502.7	([4])	26,666	54			
	39	...do	21	8⅜	526.2	([4])	43,810	83	Pile very close grained.		
	40	...do	21	7¾	486.9	([4])	34,285	70	Do.		
	41	...do	21	9	565.5	([4])	26,984	48			
	42	...do	21	9	565.5	([4])	17,460	31	Dial erratic, may have slipped.		
	43	...do	21	8¼	518.4	([4])	25,397	49			
								60.8	Average, excluding No. 42.		
8	52	...do	21	9¼	581.2	67.2	14,922	25	Concrete placed under water.		
	53	...do	21				([5])	([5])			
	54	...do	21	8¼	518.4	53.5	11,114	22	Do.		
	55	...do	21	9¾	612.6	74.7	12,700	21	Do.		
	56	...do	21	8½	534.1	56.7	15,140	28	Do.		
	59	...do	21	8⅛	510.5	51.8	13,650	27	Do.		
								25	Average.		

[1] One side of concrete block was porous and air was sucked as pile was pulled.

[2] Edge of pile caught on lower ring. (Fig. 5, point D.)

[3] 6-inch railroad spikes ⅝ by 1⅟₁₆ inch in cross section were driven 4 inches into the pile, leaving 2 inches projecting. These spikes bent as the pile was pulled and tore their way through the pile. The spikes were loose in the concrete when the pile was removed.

[4] No end bond because of plate placed between end of pile and concrete.

[5] Concrete crumbled before pulling.

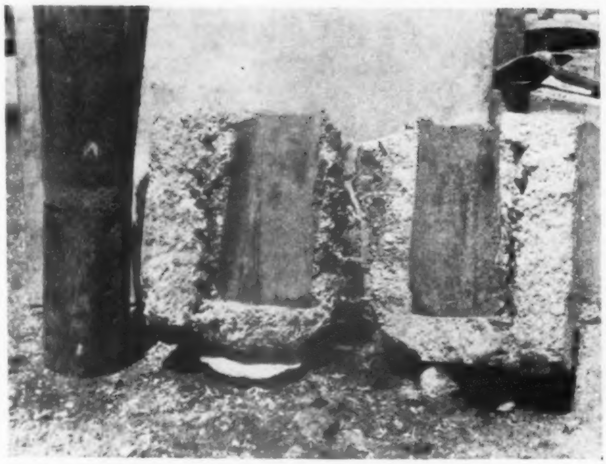
TABLE 2.—*Data pertaining to the anchorage tests*

Series	Specimen No.	Anchorage	Age	Load at initial slip	Maximum load	Slip at maximum load	Remarks
			Days	*Pounds*	*Pounds*	*Inch*	
4 A	3	1½-inch notched bar driven 27 inches in 1⅝-inch hole.	7	12,000	12,000	-------	Bar driven before casting, pile split along side.
	2	-----do-----	7	15,370	15,370	$\frac{1}{16}$	Bar driven before casting.
	1	-----do-----	21	16,880	16,880	-------	Bar driven 14 days after casting.
	4	-----do-----	21	19,200	19,200	-------	Do.
4 B	48	1½-inch plain bar driven 27 inches in 1⅝-inch hole.	21	26,120	26,120	-------	Do.
	49	-----do-----	21	19,000	19,000	-------	Bar driven 20 days after casting.
	50	-----do-----	21	13,000	13,000	-------	Do.
	51	-----do-----	21	23,750	23,750	-------	Bar driven 14 days after casting.
5	5	1½-inch fox bolt driven 27 inches in 1⅝-inch hole.	21	16,000	17,500	⅞	Do.
	6	-----do-----	21	17,430	18,160	1	Do.
	7	-----do-----	21	17,160	17,420	¾	Do.
	8	-----do-----	21	17,240	19,480	½	Do.
6	18	2 ¾-inch bars hooked under collar 4½ inches wide, pile spread with 2-inch wedges.	21	20,000	46,050	⅞	Specimen opened and showed no slip of collar.
	17	-----do-----	21	24,500	43,130	⅞	
	15	-----do-----	21	34,100	47,790	⅞	
	16	-----do-----	21	32,000	46,800	1	

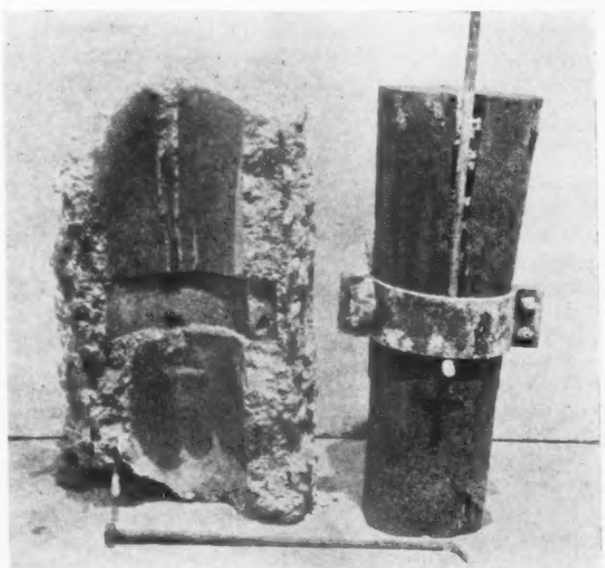
unit loads as in those tests where no such attempt was made. The average unit bond of comparable specimens in Series 1 was 62.6 pounds per square inch as compared with 64.9 and 64 pounds per square inch for Series 2 and Series 3, respectively. It is interesting and perhaps significant that the variations of the unit bond values of individual specimens of Series 2 from the average value for the series is smaller than those of any other series. In Series 3 there is no apparent increase in holding power through the use of projecting spikes nor is there any apparent difference due to the different number of spikes in the various specimens.

During the tests of Series 1 the point was raised as to whether or not the area of the end of the pile should be added to that of the embedded cylindrical surface in calculating the unit bond resistance developed. This was important since in these tests the end area was 10 per cent or more of the side area. Series 7 was added to the program to secure information on this point. In this series all possible end bond was avoided by the use of a thin metal plate as previously described. The average unit bond developed by this series is 60.8 pounds per square inch as compared with 62.6 for the comparable group of Series 1, both being computed on the basis of side area only. It appears, therefore, that the effective bond of a pile head is due to side area bond only.

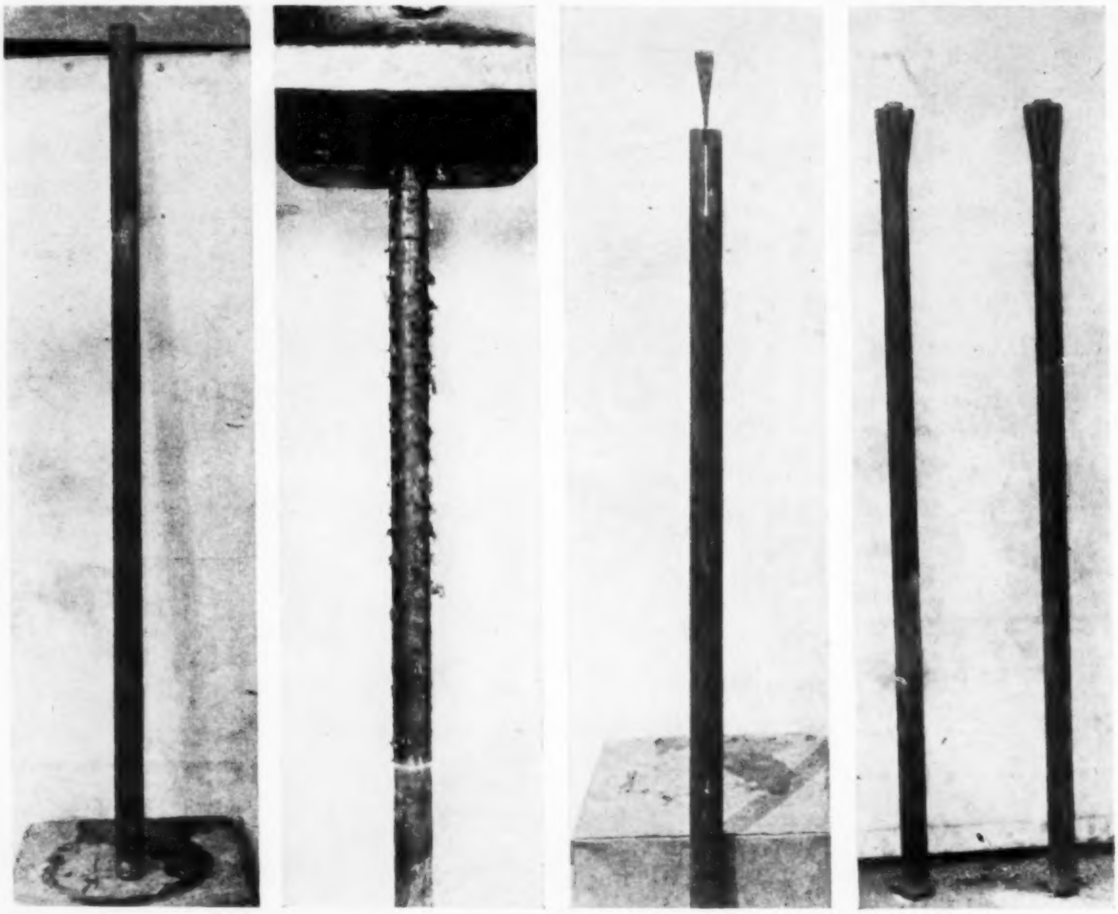
In Series 8 the concrete encasement was placed under water with a small tremie in an attempt to secure a comparison of bond values for underwater construction. It is doubtful, however, if the concrete in these specimens was of as good quality as normally would be obtained in actual construction since the pressure heads were very small. It is felt that, although the comparison of Series 8 with Series 1 indicates that much greater bond strengths were developed when the concrete was cast in air than when it was cast under



SPECIMENS OF SERIES 2 (ABOVE) AND OF SERIES 3,A (BELOW) AFTER TEST. CONCRETE CASING IS CUT OPEN TO SHOW RESULTS



A SPECIMEN OF SERIES 6 AFTER TEST. CONCRETE BLOCK CUT OPEN TO SHOW RESULTS. THE LOWER END OF THIS PILE WAS PREPARED FOR A BOND TEST AND THE CUT SHOWN WAS NOT FOR USE IN THE ANCHORAGE TEST



Pictures Taken in Order from the Left Show a Hacked Bar Used in Series 4,A Before Driving; One of These Bars Pulled from a Pile with Wood Fiber on Hacks; a Fox Bolt of Series 5 Before Driving, and Two of These Bars Pulled from a Pile Head Showing Complete Expansion of the Ends

water, too great reliance should not be placed on the values obtained in Series 8. It is thought probable that they do not represent the maximum bond which can be obtained in foundation seal construction.

ANCHORAGE METHODS PROVE EFFECTIVE

A study of the data obtained in the anchorage tests indicates that appreciable anchorage of the steel bars in the wooden pile sections was obtained with every method used in these tests. Considering first the smooth round bars of Series 4,B, it will be noted that where the bars were driven 14 days after the concrete was cast (or 7 days before testing) anchorage values of 23,750 and 26,120 pounds were obtained. Where the bars were driven 20 days after the concrete was cast (or 1 day before testing) anchorage values of 13,000 and 19,000 pounds were obtained. As the embedded area of the bar was about 127 square inches the unit values corresponding to the above totals are 187 and 205 pounds per square inch, respectively, for the first case and 102 and 150 pounds per square inch, respectively, for the second.

For the bars which had been hacked with a cold chisel (Series 4,A) the total anchorage values were 12,000 and 15,370 pounds for the two tests at the age of 7 days and 16,880 and 19,200 pounds for the tests at the age of 21 days.

In Series 5 where fox bolts were used the anchorages for the four specimens were very uniform, the average value being 16,960 pounds and the minimum value being 16,000 pounds.

The loads given as anchorage values are the maximum loads which were on the bars at the instant that the first slip was noted and obviously are not safe anchorage values.

Series 6 differs considerably from the other anchorage tests since the steel bars were hooked under a steel collar around the expanded pile section and when the casing was cast this collar was itself embedded in concrete. When a load was applied to the two projecting rods as illustrated on page 172, the stress in the bars was transferred directly to the concrete. Since the pile head reacted against the top of the testing machine this stress in the concrete was transferred to the pile section through its surface bond. The magnitude of the anchorage available depended directly on the bond between the concrete and the pile. Therefore, in view of the results of the bond tests of Series 2, it is doubtful if any useful purpose was served by the expanded pile head. The four specimens of this series showed an average anchorage of 27,650 pounds and a minimum value of 20,000 pounds. In the case of the weakest specimen a tension of more than 22,000 pounds per

(Continued on p. 183)

STRENGTH CHARACTERISTICS OF CONCRETE

RESULTS OF STUDY OF MODULUS OF ELASTICITY, EFFECT OF MOISTURE ON STRENGTH, AND BEHAVIOR UNDER REPEATED LOADING

By A. N. JOHNSON, Dean of Engineering College, University of Maryland

ARTICLES in the two preceding issues of Public Roads have discussed various phases of the general investigation of the strength characteristics of concrete conducted by the engineering experiment station of the University of Maryland with the cooperation of the United States Bureau of Public Roads and the Maryland State Roads Commission. The last article gave the results of a number of determinations of the modulus of elasticity of cores drilled from concrete roads.

This phase of the investigation was continued by making similar determinations with the same apparatus on concrete and mortar cylinders cast in the laboratory. These cylinders were 9 inches high and 4½ inches in diameter. The molds used for making the cylinders were cut to the proper length from seamless steel tubing. A slit was then made in the cylinders along an element and lugs were bolted to the cylinders so that the edges made by the slit could be held together with bolts. The cylinders were placed in a vise and the edges of the longitudinal crack forced together before the bolts on the lugs were tightened in order to avoid stripping the bolt threads. In casting a specimen the cylinders were oiled on the inside, placed on an oiled slab of slate having a smooth surface, and filled. They were then covered with a piece of oiled plate glass. Before testing, a thin plaster of Paris coating was usually placed on each end of the test cylinder and covered with a piece of plate glass to insure a flat end surface.

The cylinders were left in the molds for 24 hours. They were then removed by loosening the bolts, when the steel shell would spring open, releasing the concrete specimen. The specimens for test at ages of 30 days or less were stored in damp sand until the time of test. Those for test at later ages were stored in damp sand for about one month and then stored on shelves in the laboratory. The methods of test and gauge length were identical with those described in the preceding article.

MODULUS OF ELASTICITY DETERMINED FOR CONCRETE AND MORTAR SPECIMENS

Tables 1 and 2 give the results obtained. The mortar specimens were of 1 part cement to 2 parts sand and the average crushing strength for all ages from 7 days to 2 years was 3,563 pounds per square inch and the average modulus of elasticity was 3,381,000 pounds per square inch.

The concrete specimens were a 1:2:3 mix, the coarse aggregate being crushed limestone with a maximum size of about three-fourths inch. The average crushing strength at ages from 7 days to 2 years was 2,235 pounds per square inch and the average for the modulus of elasticity was 3,407,000 pounds per square inch.

Typical load-deformation curves for mortar specimens and for concrete specimens are shown in Figures 1 and 2. They all show a straight-line relationship for

TABLE 1.—*Crushing strength and modulus of elasticity of morta cylinders*

1:2 MORTAR, 30 DAYS OR LESS IN AGE

Specimen No.	Age	Crushing strength	Modulus of elasticity	Specimen No.	Age	Crushing strength	Modulus of elasticity
	Days	*Lbs. per sq. in.*	*Lbs. per sq. in.*		*Days*	*Lbs. per sq. in.*	*Lbs. per sq. in.*
321	7	2,180	2,730,000	335	21	3,280	3,320,000
323	7	2,420	2,760,000	336	21	2,500	3,240,000
318	8	2,370	2,760,000	337	21	2,600	3,140,000
320	8	2,610	2,860,000	360	30	2,760	3,320,000
325	10	3,000	3,280,000	362	30	2,830	3,400,000
326	10	3,230	3,280,000				
327	14	3,175	3,520,000	Average		2,842	3,165,000
328	14	3,540	3,520,000				
333	21	3,300	3,180,000				

1:2 MORTAR, OVER 30 DAYS IN AGE

Specimen No.	Age	Crushing strength	Modulus of elasticity	Specimen No.	Age	Crushing strength	Modulus of elasticity
	Years				*Years*		
183	1/7	5,200	3,360,000	58	1⅙	3,900	3,700,000
184	1/7	4,000	3,100,000	53	1¼	3,300	2,700,000
185	1/7	4,300	3,260,000	55	1¼	3,000	2,620,000
54	⅙	2,400	3,370,000	71	2	4,650	4,240,000
57	⅙	3,200	4,200,000	72	2	3,460	3,720,000
63	⅙	3,700	3,660,000	73	2	3,760	3,800,000
65	⅙	3,300	3,640,000	74	2	3,540	3,280,000
67	⅙	3,100	3,400,000	75	2	3,460	3,400,000
69	⅙	3,500	4,000,000	77	2	4,150	3,640,000
130	⅙	4,400	3,940,000	78	2	4,200	3,440,000
131	⅙	3,400	3,480,000	79	2	4,050	3,260,000
132	⅙	4,900	3,640,000	80	2	3,100	3,320,000
29	1	4,400	3,300,000	81	2	4,740	3,480,000
30	1	3,900	3,160,000	82	2	3,620	3,640,000
31	1	3,600	5,070,000	83	2	4,700	3,240,000
32	1	4,300	3,600,000	84	2	5,300	3,580,000
33	1	3,100	3,520,000	85	2	5,500	3,780,000
34	1	2,800	3,380,000	95	2	4,420	3,740,000
37	1	3,000	3,060,000	96	2	3,140	3,040,000
38	1	2,400	2,960,000	97	2	3,580	3,520,000
39	1	3,100	2,780,000	98	2	2,450	3,300,000
40	1	3,000	3,300,000	99	2	5,570	3,760,000
41	1	3,400	3,200,000	100	2	2,775	2,720,000
42	1	3,800	3,700,000	88	3	5,300	3,700,000
43	1	2,800	2,440,000				
44	1	3,300	3,300,000	Average		3,754	3,438,000
56	1	3,400	3,040,000	Grand av		3,563	3,381,000
61	1	4,200	3,320,000				
68	1	3,400	3,440,000				

the first part of the curve but gradually depart from a straight line with the higher loads. These curves were used in determining approximately the limit of proportionality or elastic limit. Curves are shown for each class of specimen; one for the higher values, one for the medium values, and one for the lower values of modulus of elasticity.

The behavior of the laboratory specimens was, in general, similar to that of cores drilled from concrete roads (described in preceding article) and substantiates the general conclusion as to the elastic behavior of concrete under pressure. Practically all of the curves for both road cores and laboratory-made specimens show the characteristic load-deformation curve as substantially a straight line for the lower portion, then a smooth curve tangent to the initial straight-line portion of the curve.

A few specimens of neat cement and of 1:1 mortar were made, and the results of tests are given in Table 2.

Table 2.—*Crushing strength and modulus of elasticity of concrete cylinders*

1:2:3 CONCRETE, 30 DAYS OR LESS IN AGE

Specimen No.	Age	Crushing strength	Modulus of elasticity	Specimen No.	Age	Crushing strength	Modulus of elasticity
	Days	*Lbs. per sq. in.*	*Lbs. per sq. in.*		*Days*	*Lbs. per sq. in.*	*Lbs. per sq. in.*
429	7	1,400	3,720,000	421	21	1,900	4,400,000
430	7	1,445	3,680,000	422	21	1,830	4,000,000
367	8	1,435	3,400,000	432	29	1,705	3,680,000
368	8	1,470	3,740,000	433	29	1,900	3,900,000
405	14	1,680	3,800,000	434	29	1,600	3,560,000
406	14	1,805	3,640,000	381	30	1,825	4,180,000
414	14	1,445	3,920,000	383	30	1,965	4,280,000
415	14	1,420	3,560,000				
416	14	1,600	3,920,000	Average		1,649	3,840,000
420	21	1,600	3,900,000				

1:2:3 CONCRETE, OVER 30 DAYS IN AGE

Specimen No.	Age	Crushing strength	Modulus of elasticity	Specimen No.	Age	Crushing strength	Modulus of elasticity
	Years				*Years*		
177	1/4	3,000	3,740,000	115	2	1,780	2,410,000
178	1/4	2,900	3,800,000	116	2	1,550	2,420,000
179	1/4	2,400	3,660,000	117	2	1,640	2,960,000
102	1	2,500	3,060,000	118	2	1,310	3,480,000
104	1	2,500	3,300,000	119	2	3,900	3,480,000
105	2	1,750	2,400,000	120	2	3,280	2,740,000
106	2	3,700	4,140,000	121	2	4,400	3,880,000
107	2	2,500	2,940,000	122	2	3,100	2,810,000
108	2	2,300	3,100,000	123	2	3,000	2,520,000
109	2	3,100	3,160,000	124	2	3,100	3,120,000
110	2	2,170	2,360,000	164	2	3,000	3,180,000
111	2	2,500	3,200,000				
112	2	2,270	3,580,000	Average		2,618	3,124,000
113	2	2,640	2,940,000	Grand av		2,235	3,407,000
114	2	1,780	2,840,000				

1:2:4 CONCRETE

Specimen No.	Age	Crushing strength	Modulus of elasticity
	Year	*Lbs. per sq. in.*	*Lbs. per sq. in.*
154	1/2	2,600	3,760,000
155	1/2	2,900	3,640,000
156	1/2	2,300	3,540,000
Average		2,600	3,646,000

NEAT CEMENT MORTAR

Specimen No.	Age	Crushing strength	Modulus of elasticity
174	2/3	5,220	2,110,000
175	2/3	3,500	1,280,000
176	2/3	3,650	1,430,000
Average		4,123	1,606,000

1:1 MORTAR

Specimen No.	Age	Crushing strength	Modulus of elasticity	Specimen No.	Age	Crushing strength	Modulus of elasticity
	Year	*Lbs. per sq. in.*	*Lbs. per sq. in.*		*Year*	*Lbs. per sq. in.*	*Lbs. per sq. in.*
46	1	2,600	3,000,000	51	1	4,300	3,000,000
48	1	2,600	3,400,000	52	1	4,000	2,800,000
49	1	3,500	3,720,000				
50	1	3,700	5,350,000	Average		3,450	3,545,000

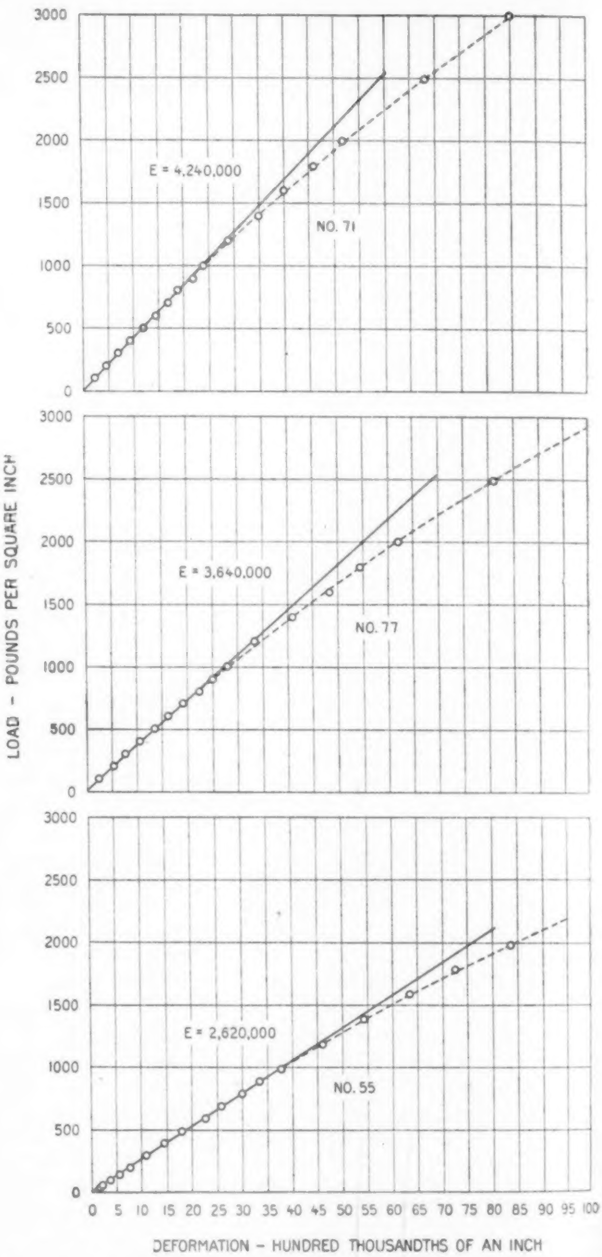


Fig. 1.—Typical Load-Deformation Curves for Mortar Specimens. Deformations Shown Are Per Inch of Length

MODULUS OF ELASTICITY AFFECTED BY AGE OF CONCRETE

Referrring to Table 1, it is seen that the mortar specimens of 30 days and less in age had an average strength of 2,842 pounds, while the specimens of 1 or 2 years age (including a few 60-day specimens) showed an average strength of 3,754 pounds per square inch. The modulus of elasticity for the 30-day specimens was 3,165,000 pounds per square inch and for the older specimens 3,438,000 pounds per square inch, a marked increase in strength and some increase in the value of the modulus of elasticity.

The concrete specimens 30 days and less in age averaged 1,649 pounds per square inch crushing strength and those 1 and 2 years old 2,618 pounds per square inch. The modulus of elasticity of the concrete 30 days or less in age was 3,840,000 pounds per square inch, while the modulus for the older specimens was 3,124,000 pounds per square inch, a marked decrease in the value for the modulus of elasticity.

The results for the concrete run parallel with the trend of the values for the modulus of elasticity found for the road cores; that is, there was apparently a dropping off in the value of the modulus of elasticity with the age of the concrete, although not as marked as for the cylinders made in the laboratory.

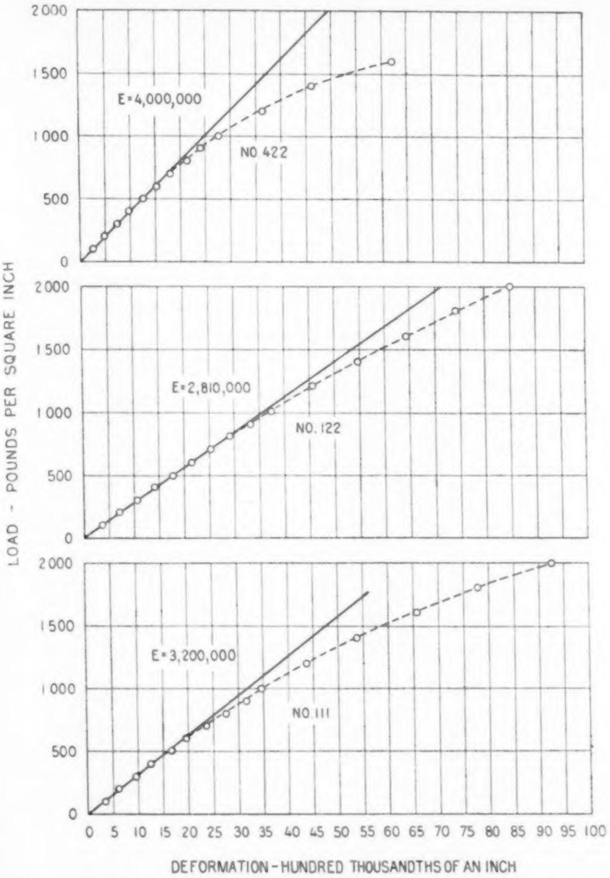


Fig. 2.—Typical Load-Deformation Curves for Concrete Specimens. Deformations Shown Are Per Inch of Length

TESTS MADE OF DRY AND MOIST CONCRETE

A number of cylinders were cast for the purpose of making comparative tests on wet and dry concrete. These cylinders were cast in groups of 10. Three such groups were made of 1 : 2 mortar and two groups were made of 1 : 2 : 3 concrete using a limestone coarse aggregate. These cylinders were cured in the same manner as other cylinders for about 24 months. At that time five cylinders from each group were immersed in water and a like number were allowed to remain in the air. The cylinders placed in water were removed after one month and together with the cylinders left in air were tested for crushing strength and modulus of elasticity. The results are shown in Table 3, which is arranged to show the grouping of the specimens—thus, 71, 73, 75, 72, and 74 were all made at the same time.

A further comparison of strength of concrete when wet compared with dry condition was made upon a number of concrete beams. These beams were originally 30 inches long, 4 inches wide, and 3 inches deep, and had been tested by cross breaking at the center. Thus, each beam furnished two short beams, one of which was submerged in water before testing. The breaking load for a 12-inch span of the beam was obtained.

The wet beams were immersed in water for about one week. The average load sustained by the dry sections of the beams for 60 samples was 1,717 pounds, while for the wet sections the average was 1,273 pounds. These beams were approximately 3 years old, although some of the specimens were but 2 years old and a few 1 year old.

Table 3.—*Effect of moisture on strength and modulus of elasticity of concrete*

Specimen No.	Dry specimen: Crushing strength	Dry specimen: Modulus of elasticity	Specimen No.	Wet specimen: Crushing strength	Wet specimen: Modulus of elasticity
1:2 MORTAR					
	Lbs. per sq. in.	*Lbs. per sq. in.*		*Lbs. per sq. in.*	*Lbs. per sq. in.*
71	4,650	4,240,000	72	3,460	3,720,000
73	3,760	3,800,000	74	3,540	3,280,000
75	3,460	3,400,000			
77	4,150	3,640,000	78	4,200	3,440,000
79	4,050	3,260,000	80	3,100	3,320,000
81	4,740	3,480,000	82	3,620	3,640,000
95	4,420	3,740,000	96	3,140	3,040,000
97	3,580	3,520,000	98	2,450	3,300,000
99	5,570	3,760,000	100	2,775	2,720,000
Average	4,264	3,649,000	Average	3,286	3,307,500
1:2:3 CONCRETE					
107	2,500	2,940,000	108	2,300	3,100,000
109	3,100	3,160,000	110	2,170	2,360,000
111	2,500	3,200,000	112	2,270	3,580,000
113	2,640	2,940,000	114	1,780	2,840,000
115	1,780	2,410,000	116	1,550	2,420,000
117	1,640	2,960,000	118	1,310	3,480,000
Average	2,360	2,935,000	Average	1,897	2,963,000

The strength of the wet specimens expressed as a percentage of the strength of the dry specimens was for the mortar cylinders 73 per cent, for the concrete specimens 80 per cent, and for the concrete beams under cross-breaking load 74 per cent. The modulus of elasticity was not particularly affected, being practically the same whether the concrete was wet or dry.

CONCRETE TESTED UNDER REPEATED LOADINGS

A number of tests were made to study the elastic behavior of concrete that had been considerably overloaded, then unloaded, and subsequently reloaded. These tests were made on concrete specimens mixed in the proportions 1:2:3 and also a number of mortar specimens mixed in the proportions of 1:2. The age of the specimens was about 2 years at the beginning of the test, which covered a period of about 1 year.

The procedure followed was to place a cylinder in the testing machine with the mirror extensometers attached and take readings corresponding to load increments of 50 to 100 pounds per square inch, the loading being carried from zero to a point well beyond the limit of proportionality or elastic limit. The load was then released, the reading of the extensometers at the zero loading noted and readings continued at short intervals for about 15 minutes, or until there was substantially no further recovery recorded. Then the specimen was submitted to a second loading under the same methods of procedure. The loading was applied four times to most of the specimens which were then laid aside and about a year later they were again submitted to a similar test, the loading being carried to the point of failure on the first run.

These experiments were made upon 5 cylinders of 1:2 mortar, 7 cylinders of 1:2:3 concrete, 3 neat-cement cylinders, and 3 road cores. Figures 3 and 4

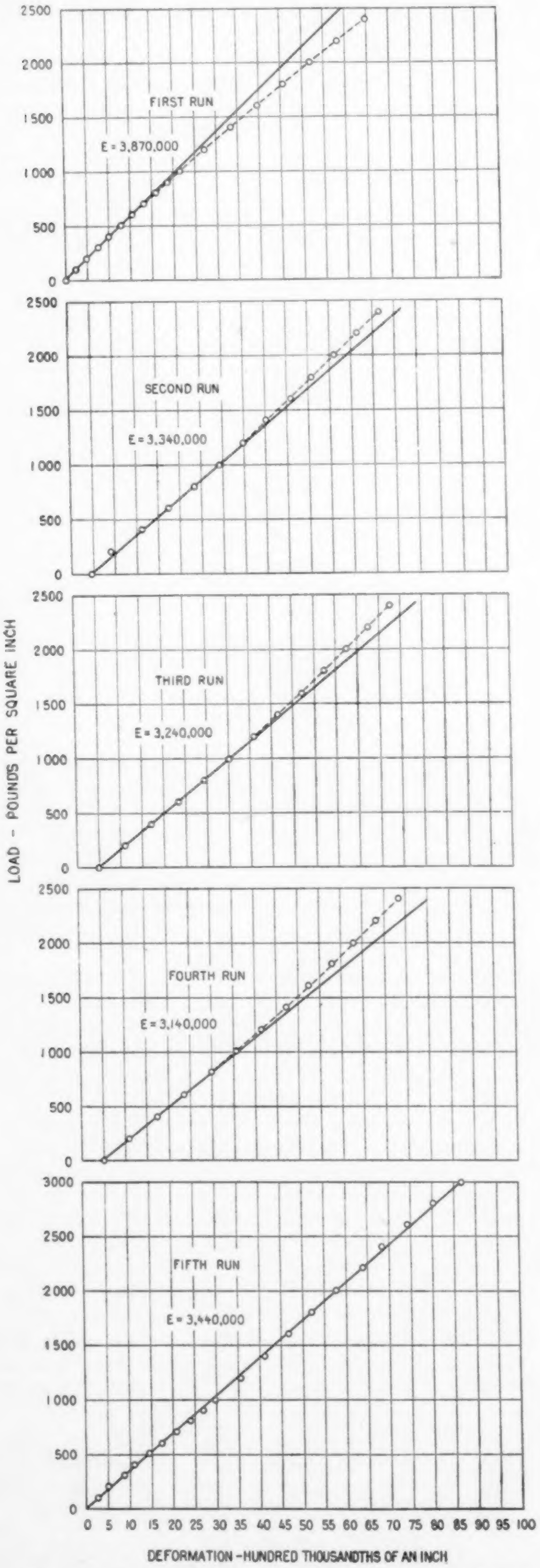


FIG. 3.—LOAD-DEFORMATION CURVES OF A 1:2:3 CONCRETE CYLINDER UNDER REPEATED LOADING

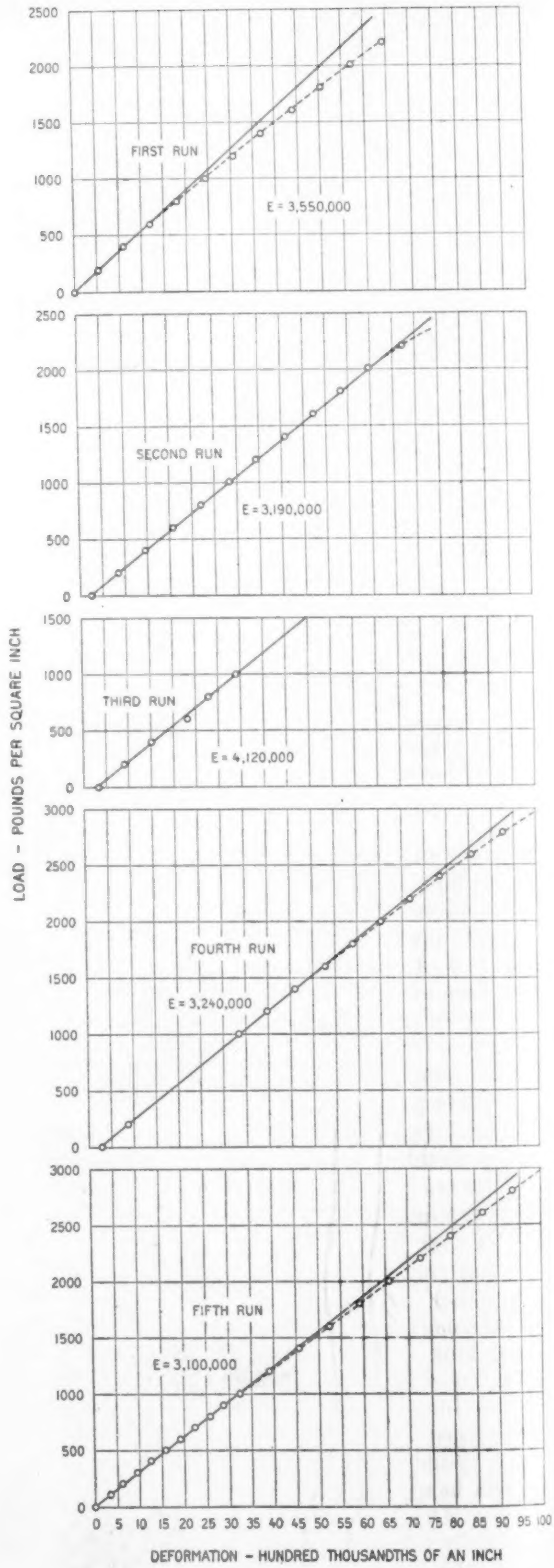


FIG. 4.—LOAD-DEFORMATION CURVES OF A 1:2 MORTAR CYLINDER UNDER REPEATED LOADING

show the results in detail for one typical mortar and one concrete specimen and present some interesting phenomena. Figure 3 shows graphically the results of tests on specimen No. 121 as described above. The deformations recorded are shown in Table 4. The load-deformation curve for the first load applications shows the ordinary characteristics that have been found for practically all the concrete and mortar specimens. It is noted first that the initial portion of each of the curves of Figure 3 is substantially a straight line. The slope of the straight-line portion of these curves measures the modulus of elasticity, which for the first run was 3,870,000 pounds per square inch; for the second run, 3,340,000 pounds per square inch; for the third run, 3,240,000 pounds per square inch; and for the fourth run 3,140,000 pounds per square inch, while for the fifth run, made a year later, the value is 3,440,000 pounds per square inch.

TABLE 4.—*Deformations of specimen No. 121 under four repeated loadings on same day*

[All deformations are recorded from zero reading at beginning of test]

	Deformation under load of 2,400 pounds per square inch	Deformation after removal of load and allowing several minutes for recovery
	Inches per inch	*Inches per inch*
First run	0.000698	0.000043
Second run	.000715	.000049
Third run	.000726	.000049
Fourth run	.000733	.000056

The elastic limit as disclosed by these diagrams (fig. 3) is about 600 pounds per square inch for the first curve, 1,100 pounds per square inch for the second and third, and for the fourth 800 pounds per square inch. For the fifth run the straight-line relationship extends well toward 3,000 pounds per square inch, the specimen breaking at 4,400 pounds per square inch. It is to be noted that for the second, third, and fourth application of the load to specimen 121 that the load-deformation curves are convex to the horizontal axis.

Figure 4 shows the curves for a similar succession of load applications to a mortar cylinder which is typical of results secured with other specimens. The slope of the curve representing the first application of load gives a value for E of 3,550,000 pounds per square inch. For the second application the value is 3,190,000 pounds per square inch; for the third application of load, 4,120,000 pounds per square inch. For the fourth it is 3,240,000 pounds per square inch; and for the fifth run it is 3,100,000 pounds per square inch.

On the first run the elastic limit is about 600 pounds per square inch. On the second application of load this limit is raised to about 2,000 pounds per square inch. The loading was not carried far enough on the third run to give an indication, but on the fourth run it was in the neighborhood of 1,500 pounds per square inch, while for the final run, after the specimen had rested for a year, the elastic limit appears to be about 1,000 pounds per square inch.

In general, the effect of overloading concrete temporarily, and then having the load released, is to reduce somewhat the value of the modulus of elasticity. The specimens suffer a deformation which is not wholly recovered and the apparent elastic limit is raised.

From the few data at hand no quantitative conclusions are warranted, but they do show that concrete behaves in a manner similar to other materials, particularly steel, which exhibit both elastic and plastic properties.

MODEL ANALYSIS OF YADKIN RIVER BRIDGE COMPLETED

In connection with the study of loading tests made on the spandrel-arch concrete bridge over the Yadkin River between Albemarle and Mount Gilead, N. C., an analysis of a celluloid model of this bridge using Beggs deformeter gauges has been made as a cooperative project by the Bureau of Public Roads and Johns Hopkins University.

In designing an open-spandrel rib arch of the type of the Yadkin River bridge it is usually assumed that the action of the rib is unaffected by the superstructure. Obviously, this is not the case, but a mathematical analysis of the complete arch, including the superstructure, is so complex as to be impracticable for purposes of design. A comparison of results from the model analysis and the measured results obtained by loading the bridge itself will indicate to what extent the action of a model made of a uniform elastic material such as celluloid may be taken as representing the action of a reinforced concrete structure which is a nonuniformly elastic material.

It is anticipated that the report on the Yadkin River bridge tests and also the report on the model studies which will be made separately will be made available in this magazine at an early date.

A MECHANICAL TRAFFIC COUNTER DEVELOPED IN DENMARK

A mechanical traffic counter has been constructed on the experimetal road at Roskildevej near Copenhagen, Denmark, which may be of interest to American engineers. This apparatus is illustrated in Figure 1. A rectangular channel with concrete bottom and sides is constructed across the roadway. This channel is covered by two substantially buit wooden gates or flaps which are approximately level with and a part of the road surface. One side of each flap rests upon the side wall of the concrete channel while the other sides meet at the center and bear upon a steel channel which is supported by three spring-supported plungers. Details of this apparatus are shown in Figure 1.

The movement of the flaps under a load is about 2 centimeters (0.8 inch) before they are brought to rest. The spring restores the flap to its initial position after the load is removed. This movement closes an electrical contact, allowing current to flow which actuates a counter. Details as to the type of counter used are not given.

In the particular installation described difficulty was encountered from horse-drawn traffic since a record was made by the horse and both axles of the vehicle. This was overcome by installing a mercury switch and timer which disconnected the current to the counter for five seconds after each contact.

The counters are read once each day and have proven very satisfactory in service. They indicate only the total number of vehicles which pass, and it is therefore necessary to make occasional counts, indicating the various classes of vehicles, for use in interpreting this data.

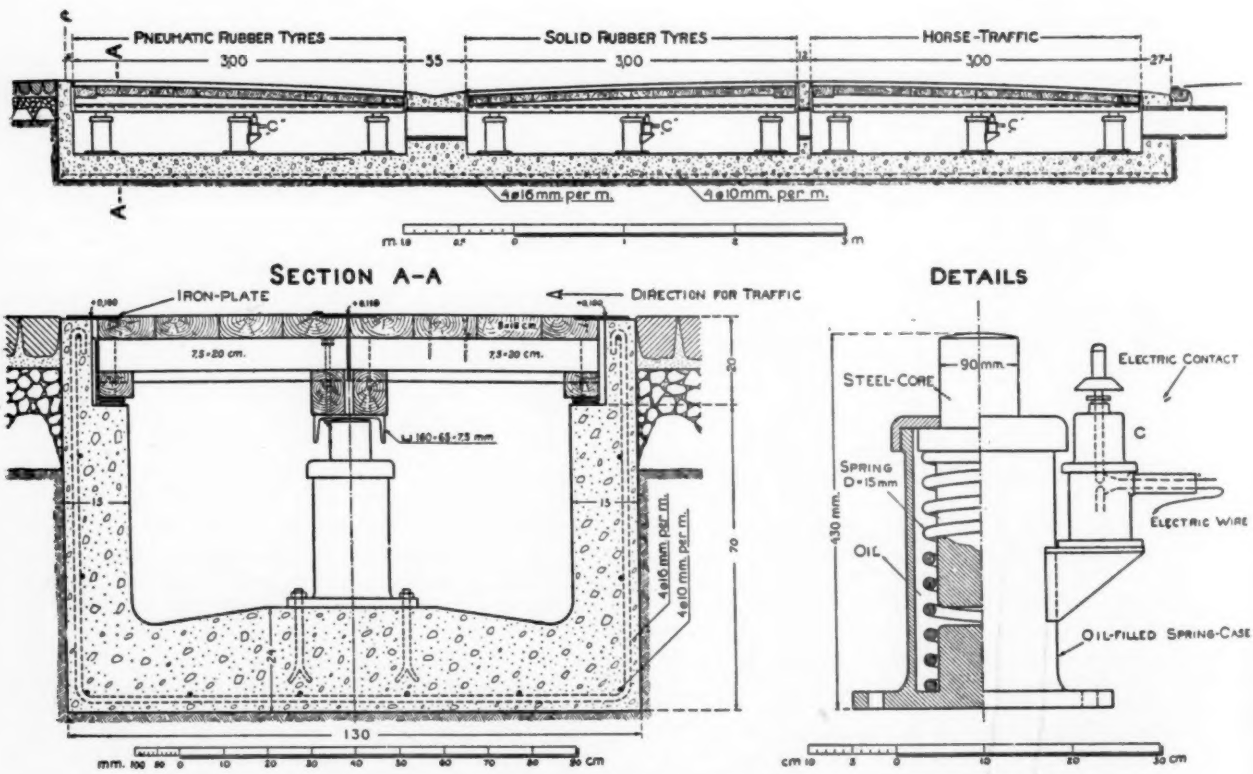


FIG. 1.—ARRANGEMENT FOR COUNTING TRAFFIC ON THREE LANES OF EXPERIMENTAL ROAD

(Continued from p. 176)

square inch was applied to the bars without slip, and failure finally occurred in each case by one of the two bars elongating and breaking in tension at the point where it was bent under the steel collar. The concrete casings were broken open after the tests and showed neither movement of the collar on the pile section nor of the pile section in the concrete casing.

CONCLUSIONS PRESENTED

From the analysis of the data obtained in these tests the following general conclusions seem to be justified:

BOND TESTS

Definite and appreciable bond can be developed between timber pile heads and the concrete in which they are encased.

The bond between the pile head and the surrounding concrete appears to increase between the ages of 7 and 21 days.

Better strength concrete and much greater bond were obtained where the casting of the concrete casings took place in air than where it was done under water. Particular attention is called to the fact that the concrete placed under water in these tests was probably not representative of the best obtainable in actual foundation seal construction, as pointed out in the foregoing discussion.

The total bond developed between the pile head and the surrounding concrete is a function of the area of the embedded cylindrical surface of the pile. The bond at the end of the pile appears to be negligible.

Pile sections that had been peeled and roughly trimmed developed, in general, as high bond strength as did those where the attempt was made to increase the holding power artificially by any of the methods included in these experiments.

ANCHORAGE TESTS

Appreciable anchorage of steel bars in the ends of wood pile heads may be obtained by any of the methods used in these tests.

Plain round reinforcing bars driven into undersize holes appear to develop as great an anchorage as either the hacked bars or the fox bolts, although the anchorage obtained with the fox bolts seems to be somewhat more consistent than that obtained with either of the other two methods.

The methods of anchoring steel bars to wood pile heads encased in concrete by means of a steel collar to which the bars are attached may be depended upon to furnish greater anchorage than any of the methods mentioned above. However, attention is called to the fact that in this case the transfer of stress from the bars to the pile is through the surface bond between the concrete and the pile section. It is doubtful if either the expanded pile head or the fact that the steel collar is directly in contact with the pile appreciably affect the anchorage.

EFFECT OF MOISTURE ON TOUGHNESS OF ROCK

A recent visitor [1] to the Bureau of Public Roads, stated that in India there have been frequent failures of road metal when it was thoroughly saturated with water. No such occurrences have been reported in the United States, but it was considered of interest to make a few laboratory tests to investigate this condition. The failures were reported to have occurred under horse-drawn and animal traffic, so it was believed that the toughness of the rock was affected by absorption of water.

A number of toughness specimens were prepared from six different kinds of rock. These materials were loose-textured granular sandstone, soft crystalline limestone, soft amorphous limestone, hard siliceous limestone, trap (diabase), and granite.

One-half of the specimens of each group were immersed in water for five days to permit them to become thoroughly saturated, and were tested immediately upon removal from the water. All of the immersed specimens were found to be thoroughly saturated.

The samples were tested in the Page impact machine for toughness of rock using the standard 2-kilogram hammer.[2] The results of the tests are shown in accompanying Table 1.

TABLE 1.—*Toughness of various kinds of rock when tested in a dry and saturated condition*

Sample No.	Sandstone	Limestone	Limestone	Limestone	Diabase	Granite
TESTED IN DRY CONDITION						
1	5	5	4	12	13	8
2	5	4	4	11	12	7
3	4	4	4	12	13	7
4	5	5	4	13	13	------
5	5	4	4	12	12	------
Average	5	4	4	12	13	7
TESTED IN WET CONDITION						
1	5	5	4	12	14	6
2	4	5	4	11	12	9
3	4	5	4	13	11	7
4	5	5	4	13	11	-------
5	4	4	4	13	13	-------
Average	4	5	4	12	12	7

It will be noted that the toughness of the dry rock is the same as that when the rock is thoroughly saturated with water. This applies to all the materials tested which range from very soft to hard rock. It is therefore considered improbable that a road failure would occur in the road metal itself of the types tested due solely to absorption of water.

[1] B. A. Acharya, Engineer P. W. D., Bombay, India.
[2] A full description of this test may be found in JACKSON, FRANK H. JR., METHODS FOR THE DETERMINATION OF THE PHYSICAL PROPERTIES OF ROAD-BUILDING ROCK. U. S. Dept. Agri. Bull. No. 347, 28 p., illus., 1916.

COMPREHENSIVE CONCRETE PAVEMENT CURING TESTS NOW IN PROGRESS IN TENNESSEE

The most comprehensive series of concrete pavement curing tests yet undertaken was initiated during the past summer on Tennessee Federal-aid project 18–A between Memphis and Somerville. The Bureau of Public Roads is cooperating with the Tennessee Department of Highways and Public Works in this investigation which has been planned to include practically every curing method which has received serious consideration.

The general scheme of the test is to cure one side of the pavement continuously with the State standard method, consisting of application of wet burlap for 24 hours followed by a 2-inch earth covering kept wet for 10 days. For comparison with this standard curing, the other side of the pavement will consist of a series of sections, each consisting of a day's run or approximately 1,000 feet, and each cured in a different manner.

The sections will be as described and in the order tabulated below. It should be possible to repeat the entire series at least four times on the 17-mile project making 4,000 linear feet of pavement cured by each method.

The design of the pavement was modified from the State standard 8–6–8 cross section to 8–7–8 so as to eliminate all tie bars across the center joint which would restrict the expansion or contraction of one side of the slab with respect to the other and so effect the results of the test. The pavement consists of plain concrete, 18 feet wide, with a metal center strip from which the ¾-inch pins to the subgrade are removed as soon as possible after the pavement is laid. The earth shoulder on each side of the pavement is 4 feet wide.

The following are the curing methods used:

1. Wet burlap for 24 hours—no further curing.
2. Wet burlap for 48 hours—no further curing.
3. Wet burlap for 72 hours—no further curing.
4. Wet burlap for 96 hours—no further curing.
5. No curing whatever.
6. Sisalcraft for 24 hours—no further curing.
7. Wet burlap for 24 hours followed by surface application of sodium silicate.
8. Asphaltic emulsion applied immediately after finishing.
9. Wet burlap for 24 hours followed by surface application of calcium chloride.
10. Calcium chloride admixture and wet burlap for 24 hours.
11. Coal tar applied cold immediately after finishing.
12. Hunt process applied immediately after finishing.
13. Tar paper on subgrade. Hunt process applied immediately after finishing.
14. Tar paper on subgrade. Coal tar applied cold immediately after finishing.
15. Tar paper on subgrade. Calcium chloride admixture and wet burlap for 24 hours.
16. Tar paper on subgrade. Wet burlap for 24 hours followed by surface application of calcium chloride.
17. Tar paper on subgrade. Asphaltic emulsion applied immediately after finishing.
18. Tar paper on subgrade. Wet burlap for 24 hours followed by surface application of sodium silicate.
19. Wet burlap for 24 hours followed by dry earth curing.
20. Wet burlap for 24 hours followed by ponding.

For each experimental section it is planned to cast 24 concrete beams 6 by 6 by 46 inches in size—12 on the wet earth curing side and 12 on the experimental side. These specimens will be cast in three groups of four specimens on each side at approximately equal intervals throughout a section. The molds will be removed at the end of 24 hours and the sides of the specimens protected from drying out by means of tar paper, against which earth will be banked. The subsequent curing will be as nearly as possible like that of the adjacent pavement section. In addition there will be cast for each experimental section one set of four beams which will receive no curing whatever. Beams will be tested at the age of 3, 7, 14, and 28 days. The length of the beams is sufficient to permit a test on each beam at each of these four periods. This will give 28 tests for modulus of rupture of concrete for each age and for each test section—12 on the standard curing side, 12 on the experimental curing side, and 4 on the beams which are not cured.

Detailed observations as to temperature, humidity, wind velocity, rainfall, condition of concrete and subgrade, and other details will be made throughout the progress of the work.

Concrete cores will also be drilled from each section at points corresponding to the locations represented by the concrete beams. These cores will be used for the determination of strength of concrete at periods subsequent to 28 days.

ROAD PUBLICATIONS OF BUREAU OF PUBLIC ROADS

Applicants are urgently requested to ask only for those publications in which they are particularly interested. The Department can not undertake to supply complete sets nor to send free more than one copy of any publication to any one person. The editions of some of the publicctions are necessarily limited, and when the Department's free supply is exhausted and no funds are available for procuring additional copies, applicants are referred to the Superintendent of Documents, Government Printing Office, this city, who has them for sale at a nominal price, under the law of January 12, 1895. Those publications in this list, the Department supply of which is exhausted, can only be secured by purchase from the Superintendent of Documents, who is not authorized to furnish publications free.

ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1924.
Report of the Chief of the Bureau of Public Roads, 1925.
Report of the Chief of the Bureau of Public Roads, 1927.

DEPARTMENT BULLETINS

No. 105D. Progress Report of Experiments in Dust Prevention and Road Preservation, 1913.
*136D. Highway Bonds. 20c.
220D. Road Models.
257D. Progress Report of Experiments in Dust Prevention and Road Preservation, 1914.
*314D. Methods for the Examination of Bituminous Road Materials. 10c.
*347D. Methods for the Determination of the Physical Properties of Road-Building Rock. 10c.
*370D. The Results of Physical Tests of Road-Building Rock. 15c.
386D. Public Road Mileage and Revenues in the Middle Atlantic States, 1914.
387D. Public Road Mileage and Revenues in the Southern States, 1914.
388D. Public Road Mileage and Revenues in the New England States, 1914.
390D. Public Road Mileage and Revenues in the United States, 1914. A Summary.
407D. Progress Reports of Experiments in Dust Prevention and Road Preservation, 1915.
463D. Earth, Sand-clay, and Gravel Roads.
*532D. The Expansion and Contraction of Concrete and Concrete Roads. 10c.
*537D. The Results of Physical Tests of Road-Building Rock in 1916, Including all Compression Tests. 5c.
*583D. Reports on Experimental Convict Road Camp, Fulton County, Ga. 25c.
*660D. Highway Cost Keeping. 10c.
*670D. The Results of Physical Tests of Road-Building Rock in 1916 and 1917. 5c.
*691D. Typical Specifications for Bituminous Road Materials. 10c.
*724D. Drainage Methods and Foundations for County Roads. 20c.
*1077D. Portland Cement Concrete Roads. 15c.
1259D. Standard Specifications for Steel Highway Bridges, adopted by the American Association of State Highway Officials and approved by the Secretary of Agriculture for use in connection with Federal-aid road work.
1279D. Rural Highway Mileage, Income, and Expenditures, 1921 and 1922.

DEPARTMENT BULLETINS—Continued

No. 1486D. Highway Bridge Location.

DEPARTMENT CIRCULARS

No. 94C. T. N. T. as a Blasting Explosive.
331C. Standard Specifications for Corrugated Metal Pipe Culverts.

TECHNICAL BULLETIN

No. 55. Highway Bridge Surveys.

MISCELLANEOUS CIRCULARS

No. 62M. Standards Governing Plans, Specifications, Contract Forms, and Estimates for Federal Aid Highway Projects.
93M. Direct Production Costs of Broken Stone.
*105M. Federal Legislation Providing for Federal Aid in Highway Construction and the Construction of National Forest Roads and Trails. 5c.

FARMERS' BULLETIN

No. *338F. Macadam Roads. 5c.

SEPARATE REPRINTS FROM THE YEARBOOK

No. 914Y. Highways and Highway Transportation.
937Y. Miscellaneous Agricultural Statistics.

TRANSPORTATION SURVEY REPORTS

Report of a Survey of Transportation on the State Highway System of Connecticut.
Report of a Survey of Transportation on the State Highway System of Ohio.
Report of a Survey of Transportation on the State Highways of Vermont.
Report of a Survey of Transportation on the State Highways of New Hampshire.

REPRINTS FROM THE JOURNAL OF AGRICULTURAL RESEARCH

Vol. 5, No. 17, D- 2. Effect of Controllable Variables upon the Penetration Test for Asphalts and Asphalt Cements.
Vol. 5, No. 19, D- 3. Relation Between Properties of Hardness and Toughness of Road-Building Rock.
Vol. 5, No. 24, D- 6. A New Penetration Needle for Use in Testing Bituminous Materials.
Vol. 6, No. 6, D- 8. Tests of Three Large-Sized Reinforced-Concrete Slabs Under Concentrated Loading.
Vol. 11, No. 10, D-15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

* Department supply exhausted.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CURRENT STATUS OF FEDERAL AID ROAD CONSTRUCTION

AS OF

OCTOBER 31, 1928

STATE	COMPLETED MILEAGE	UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FEDERAL-AID FUNDS AVAILABLE FOR NEW PROJECTS	STATE
		Estimated total cost	Federal aid allotted	MILEAGE			Estimated total cost	Federal aid allotted	MILEAGE				
				Initial	Stage[1]	Total			Initial	Stage[1]	Total		
Alabama	1,824.4	$ 4,735,829.62	$ 2,363,508.63	308.0	37.9	345.9	$ 216,769.66	$ 108,384.82		19.7	19.7	$ 1,366,533.42	Alabama
Arizona	884.1	1,343,491.30	1,153,202.34	63.0	2.4	65.4	89,976.13	65,088.70	6.1	5.6	11.7	2,699,281.54	Arizona
Arkansas	1,698.9	4,804,094.71	2,147,567.01	165.7	6.2	171.9	93,302.28	38,681.12	10.9	.3	11.2	1,713,614.30	Arkansas
California	1,501.8	7,596,088.02	3,554,994.66	182.6	8.7	191.3	1,504,278.32	769,116.60	39.7		39.7	1,889,496.77	California
Colorado	986.7	5,322,047.84	2,706,741.61	184.7	31.2	215.9	711,553.48	390,759.31	35.6		35.6	1,795,977.94	Colorado
Connecticut	213.1	2,385,081.69	525,091.53	28.7		28.7	179,362.16	39,975.00	2.7		2.7	595,098.79	Connecticut
Delaware	203.2	499,461.60	178,885.54	13.0	2.1	15.1	111,232.00	40,800.00	2.7		2.7	154,677.22	Delaware
Florida	429.5	2,318,953.51	1,000,227.20	68.5	5.5	74.0	536,597.57	268,298.77	30.0		30.0	1,214,982.25	Florida
Georgia	2,424.8	6,206,233.49	2,792,181.33	257.4	63.4	320.8	446,910.80	206,034.68	22.3	8.4	30.7	14,619.27	Georgia
Idaho	1,020.6	2,496,654.98	1,488,791.81	169.7	26.2	195.9	61,000.00	35,000.00	6.3		6.3	82,925.61	Idaho
Illinois	1,743.8	21,451,944.47	9,729,285.21	666.6		666.6	1,113,020.60	556,279.18	37.2		37.2	102,147.12	Illinois
Indiana	1,135.9	9,081,617.21	4,331,065.07	275.6	3.5	279.1	1,266,814.61	540,459.39	40.2		40.2		Indiana
Iowa	2,869.1	6,363,179.89	2,700,760.89	92.3	160.2	252.5	595,767.19	231,211.52	3.8	35.0	38.8	187,554.17	Iowa
Kansas	2,283.7	5,750,998.35	2,294,464.43	321.2	12.0	333.2	2,293,694.58	875,771.63	135.9	10.9	146.8	155,549.44	Kansas
Kentucky	1,206.3	5,575,419.84	2,574,646.35	245.3		245.3	560,672.99	275,336.48	28.6		28.6	196,988.27	Kentucky
Louisiana	1,276.0	4,884,408.20	2,434,445.45	198.4		198.4	103,065.30	25,000.00	.1		.1	185,163.73	Louisiana
Maine	442.2	2,133,349.05	734,380.82	50.1		50.1	904,721.22	353,959.23	28.9		28.9	869,077.41	Maine
Maryland	557.6	1,742,202.42	799,800.00	70.3	7.2	77.5						30,546.83	Maryland
Massachusetts	531.3	4,190,602.15	1,361,265.68	82.8		82.8						1,556,809.31	Massachusetts
Michigan	1,390.5	12,847,446.17	5,420,740.49	335.2		335.2	927,885.00	353,180.50	22.4	6.5	28.9	149,256.45	Michigan
Minnesota	3,877.6	5,731,811.18	2,020,740.93	260.1	75.5	335.6	1,302,737.46	99,000.00	28.6		28.6	29,330.50	Minnesota
Mississippi	1,591.9	4,888,680.55	2,212,697.25	195.6	31.6	227.2	725,723.52	362,862.11	45.2		45.2	176,149.96	Mississippi
Missouri	2,228.0	6,243,406.90	2,682,005.48	188.2	52.9	241.1	901,098.39	403,960.57	19.3	9.8	29.1	711,516.59	Missouri
Montana	1,455.4	3,676,455.93	2,248,330.89	284.3	16.2	300.5	845,889.36	449,373.62	89.0	1.4	90.4	4,019,130.03	Montana
Nebraska	3,349.9	4,814,194.78	2,387,303.06	456.9	106.3	563.2	404,227.60	201,568.20	16.7	41.8	58.5	1,736,585.05	Nebraska
Nevada	1,029.8	1,116,287.28	971,183.85	108.7	66.4	175.1	2,426.36	2,128.40		1.6	1.6	320,132.03	Nevada
New Hampshire	316.7	771,141.69	299,931.95	20.9		20.9	61,087.21	23,820.00	1.6		1.6	33,298.16	New Hampshire
New Jersey	433.5	5,702,303.15	978,207.35	67.2		67.2	164,227.71	44,595.00	3.0		3.0	42,170.94	New Jersey
New Mexico	1,765.2	3,541,972.04	2,300,391.30	221.4	.5	221.9	448,929.52	284,080.43	24.6	5.1	29.7	115,664.45	New Mexico
New York	1,899.0	33,609,400.00	7,705,177.50	504.7	8.6	513.3	4,946,700.00	958,756.00	63.9		63.9	3,451,909.55	New York
North Carolina	1,598.2	2,463,767.57	1,195,388.43	85.9	25.4	111.3	823,665.30	407,942.55	39.8	7.2	47.0	486,152.06	North Carolina
North Dakota	3,410.4	3,458,361.12	1,538,798.17	536.3	211.9	748.2	883,999.48	369,092.07	87.1	126.2	213.3	66,505.91	North Dakota
Ohio	1,864.2	13,880,447.01	5,080,753.85	312.6	6.0	318.6	2,623,580.80	804,621.44	43.1	12.8	55.9	1,461,485.93	Ohio
Oklahoma	1,862.8	3,475,513.57	1,600,035.82	159.8	14.3	174.1	1,159,087.40	501,869.79	55.7	16.5	72.2	110,692.21	Oklahoma
Oregon	1,121.5	1,285,337.84	735,858.41	37.3		37.3	64,043.10	39,020.13	5.5		5.5	1,227,166.69	Oregon
Pennsylvania	1,932.4	15,244,781.10	4,296,043.56	265.0		265.0	2,365,137.61	792,920.90	48.5		48.5	484,176.39	Pennsylvania
Rhode Island	159.6	278,228.17	83,475.00	5.6		5.6	402,840.59	92,334.56	4.8		4.8	535,134.49	Rhode Island
South Carolina	1,666.7	8,162,602.59	1,750,114.71	174.9	122.4	297.3	3,046.08	1,000.00	1.1		1.1	63,366.58	South Carolina
South Dakota	3,135.0	2,701,176.30	1,471,160.06	490.7	60.8	551.5	387,485.01	213,116.67	40.8	15.3	56.1	74,424.15	South Dakota
Tennessee	1,045.8	6,248,253.88	2,544,741.31	138.3	49.6	187.9	1,725,870.64	511,900.18	6.5	44.6	51.1	464,373.78	Tennessee
Texas	6,000.3	10,453,295.11	4,230,044.00	239.2	181.1	420.3	6,248,424.37	2,513,884.27	336.4	114.7	451.1	2,213,631.12	Texas
Utah	848.4	1,576,625.93	1,066,367.19	77.9	9.3	87.2	415,240.51	302,991.62	38.5	7.9	46.4	50,558.98	Utah
Vermont	225.0	1,186,576.26	361,788.03	25.0		25.0						42,081.39	Vermont
Virginia	1,275.0	4,398,271.40	1,401,161.87	113.4	21.6	135.0	981,946.61	178,222.56	33.6		33.6	11,604.44	Virginia
Washington	813.7	3,537,809.46	1,149,775.25	77.9	18.1	96.0	892,277.03	357,756.27	17.2	5.0	22.2	446,548.83	Washington
West Virginia	655.7	2,069,325.71	910,500.08	78.3	2.5	80.8	667,359.78	308,698.26	15.6	9.9	25.5	284,201.46	West Virginia
Wisconsin	2,151.0	7,269,186.77	2,570,943.29	204.2	32.7	236.9	500,769.19	248,640.00	14.8		14.8	1,169,744.49	Wisconsin
Wyoming	1,541.5	2,119,392.39	1,277,720.61	227.9	10.1	238.0						10,047.90	Wyoming
Hawaii	39.4	64,548.41	32,274.20	.1		.1	111,383.58	25,227.00	1.7		1.7	1,066,498.59	Hawaii
TOTALS	73,716.1	271,698,268.70	107,394,969.44	9,337.4	1,490.3	10,827.7	41,765,818.10	15,672,718.52	1,536.0	506.2	2,042.2	35,864,580.89	TOTALS

[1] The term stage construction refers to additional work done on projects previously improved with Federal aid. In general, such additional work consists of the construction of a surface of higher type than was provided in the initial improvement.